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Design of a Center-Bearing Deck Plate-Girder Draw-Span

Civil Engineering

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DESIGN

OF A

CENTER-BEARING DECK PLATE-GIRDER DRAW-SPAN

BY

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C O L L E G E   O F   E N G I N E E R I N G

April 30, 1907.

This is to certify that the following thesis prepared under the immediate direction of Professor F. O. Dufour, Assistant Professor of Structural Engineering, by

HARRY SPENCER PECK

entitled DESIGN OF A CENTER-BEARING DECK PLATE-GIRDER DRAW-SPAN

is accepted by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

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Head of Department of Civil Engineering

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INTRODUCTION.

Plate girders have been in use in this country for about fifty years, having been introduced from Europe where they were first made about 1850. Although the stresses in them are not exactly determinate, nevertheless the cheapness of shop cost, the stiffness, and the ease and quickness of erection give them a great advantage over truss bridges for spans from 30 to 100 feet.

The deck plate girder is particularly desirable where it can be be substituted for a through truss bridge, as the expensive floor system is thereby eliminated. This form is well adapted to draw-bridges owing to its compactness, the space available for machinery, and the ease with which the latter is attached.

This thesis is devoted to the design of a center-bearing deck plate-girder draw-span. In any such design it is necessary to have a set of specifications in order that there may be an exact understanding between the builder and the engineer, and to form a set of rules for the acceptance of the finished structure. In this design, Cooper's General Specifications for Steel Railroad Bridges and Viaducts, Edition 1901, will be followed, the uniform instead of the concentrated loading being used.

Art. 1. General Dimensions.

The lenght under coping will be taken as 34 feet for each arm, making 80 feet center to center of the end wedges and 81 feet over all. It is desirable to have a sufficient depth at the center to allow the structure to be supported by cross-girders resting on the center bearing, and also to give room for the machinery required to operate the draw. Accordingly, 6 feet is taken as the center depth. At the ends there must be space enough between the bottom of the girders and the piers to permit the wedges to be placed between them. Four feet is taken as this depth.

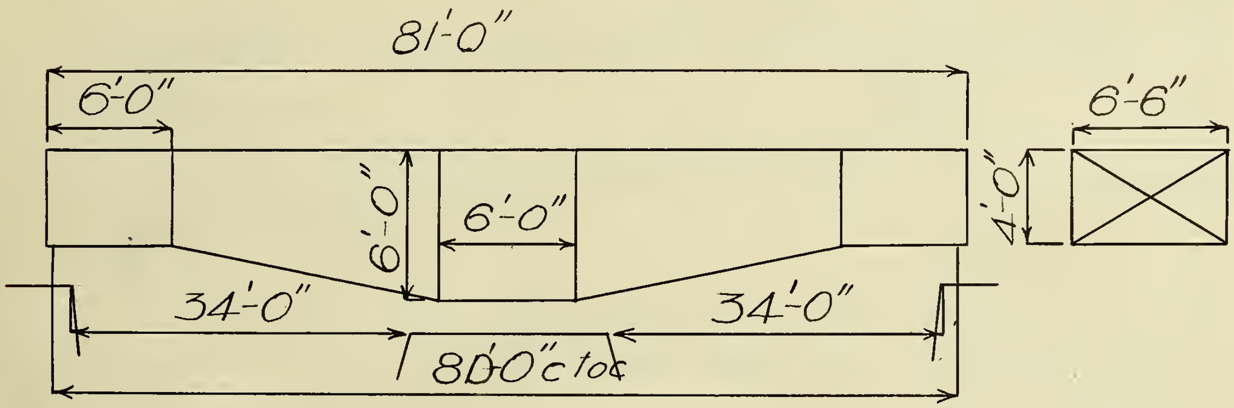


Fig. 1.

Fig. 1 is a diagram showing the various dimensions as decided upon.

Art. 2. Weight.

For the preliminary design, it is necessary to estimate the weight of the steel, and for this purpose empirical formulae have been devised. For ordinary deck plate girders designed for Cooper's E. 40 loading, $w = 124 + 10L$ is a formula giving close results. Here, w is the weight in pounds per linear foot of span, and L is the length of the span in feet. For the extra weight of the draw

it will be necessary to add 15 per cent. This gives 600 pounds per linear foot of span for the weight of the steel.

Art. 3. Loadings.

The bridge will be designed for the stresses caused under four conditions. These conditions are so chosen as to give a combination from which the maximum possible stress at any point can be determined. These conditions are:-

(a) The draw swinging, dead load only.

(b) Draw closed, and considered as two continuous spans for dead load.

(c) Draw closed, and each span considered as independent for live load.

(d) Draw closed, and considered as two continuous spans for live load.

The possible combinations are a+c, a+d, b+c, and b+d. The reactions, and the shears and moments will now be computed for these various conditions.

Art. 4. Reactions.

When the end wedges are drawn, the entire weight is carried at the center bearing. For cases (b) and (d), the reactions are given by the formulae $R_1 = R_3 = 3/8wl$, and $R_2 = 10/8wl$, where w is the load per linear foot and l is the length center to center of span. The following table, Table I, gives the results as indicated above, and are for one girder.

Table I. Reactions.

	(a)	(b)	(c)	(d)
R_1	0	7 750	75 400	50 840

Table I. Reactions. (continued)

	(a)	(b)	(c)	(d)
R_2	40 500	25 000	73 600	188 500
R_3	0	7 750	75 400	50 840

The shears and moments were computed at sections at distances apart of five feet. Fig. 2 indicates the sections at which the shears and moments were computed, and it also shows their re-

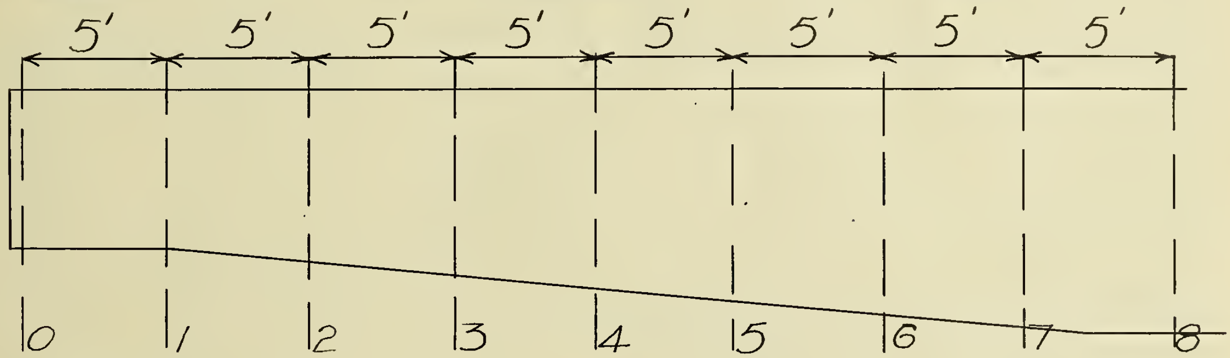


Fig. 2.

lative position on the girder.

Table II gives the shears in pounds at the five-foot sections, for the various conditions of loading.

Table II.

Shears.

Sect.	(a)	(b)	(c)	(d)
V_0	0	+ 7 595	+ 77 790	+ 58 430
V_1	- 2 750	+ 4 844	+ 72 130	+ 37 700
V_2	- 5 250	+ 2 344	+ 42 410	+ 18 850
V_3	- 7 750	- 150	+ 29 450	0
V_4	- 10 250	- 2 656	+ 18 850	- 18 850
V_5	- 12 750	- 5 156	- 29 450	- 37 700
V_6	- 15 250	- 7 656	- 42 410	- 56 550
V_7	- 17 750	- 10 156	- 72 130	- 75 400
V_8	- 20 250	- 12 650	- 75 700	- 94 250

The moments under the various conditions of loading are computed and are placed in Table III. The results are in inch-pounds.

Table III.

Moments.

Sect.	(a)	(b)	(c)	(d)
M_0	0	0	0	0
M_1	- 90 750	+ 375 000	+ 3 444 000	+ 3 052 000
M_2	- 330 750	+ 600 000	+ 5 904 000	+ 4 930 000
M_3	- 720 750	+ 675 000	+ 7 380 000	+ 5 919 000
M_4	- 1 260 750	+ 600 000	+ 7 872 000	+ 6 903 000
M_5	- 1 950 750	+ 375 000	+ 7 380 000	+ 4 935 000
M_6	- 2 790 750	+ - 0	+ 5 904 000	+ 2 970 000 - 200 000
M_7	- 3 780 750	- 525 000	+ 3 444 000	- 5 504 000
M_8	- 4 920 750	- 1 200 000	0	- 7 872 000

Art. 5. Flange Stresses and Areas.

By selecting the combinations giving the maximum moments, and dividing these moments by the depth of the girder less two inches, the maximum flange stresses are computed. They are given in Table IV. Whenever there is a reversal of stresses, eight-tenths of the smaller is added to the larger, and the result is used in designing the parts under consideration. The first three-fourths of each span has such a condition when the span is swinging, and eight-tenths of the stress in that case must be added to the dead load in computing the flange areas. The allowable unit stresses are 20 000 pounds for dead load, and 10 000 pounds for live load. By dividing the moment by these the respective net flange area results.

Table IV gives the maximum dead and live load as well as the reversal moments. Table V gives the flange areas and stresses.

Table IV. Maximum Moments.

Pt.	Live.	Dead.	Reversal.
1.	+ 3 444 000	+ 375 000	+ 73 000
2.	+ 5 904 000	+ 600 000	+ 265 000
3.	+ 7 380 000	+ 675 000	+ 577 000
4.	+ 7 872 000	+ 600 000	+ 1 009 000
5.	+ 7 380 000	+ 375 000	+ 1 561 000
6.	+ 5 904 000	0	+ 2 393 000
7.	- 5 504 000	- 3 781 000	0
8.	- 7 872 000	- 4 921 000	0

Table V. Flange Stresses and Areas.

Assumed H_e	Stresses.		Flange Areas.		
	Live.	Dead.	Live.	Dead.	Total.
46.00	74 700	9 720	7.47	0.48	7.95
49.31	119 500	17 500	11.95	0.87	12.82
53.25	138 500	23 500	13.85	1.17	15.02
57.00	137 800	28 200	13.78	1.41	15.19
60.75	123 000	31 700	12.30	1.58	13.88
64.63	91 100	36 900	9.11	1.89	11.00
68.50	80 200	55 100	8.02	2.75	10.77
70.00	112 500	70 200	11.25	3.51	14.76

PART II. DESIGN.

Art. 6. Flanges.

In the design for the flanges, two or more steps are necessary. First, the design using the net areas as computed in Table

V, and second, the investigation of this design by using the true effective depth. The make-up of the flange at the point of maximum required area will now be determined.

This section is at the center of either arm, and the required area is 15.19 square inches. The area should be about equally divided between the angles and the cover plates. By dividing 15.19 by 4, the approximate net area of one angle is obtained. The result is 3.80 square inches, and, from Cambria, a 6" x 4" x 1/2" angle is selected. This has a gross area of 4.75 square inches. Deducting two 7/8-inch rivet holes from this, gives a net area of 3.75 square inches. Deducting the net area of the angles from the total net area required gives 7.69 square inches as the required net area of the cover plates. For 4-inch angles, 10-inch plates should be used. Two plates, one 1/2" thick, and the other 7/16" thick, give a gross area of 9.38 sq. in., and a net area of 7.50 sq. in. This makes a total of 15.00 sq. in. as the net flange area given by this section. This finishes the preliminary design, and the section will now be investigated.

The first step in the investigation of the design is the determination of the center of gravity of the flanges. By taking moments about the top of the cover plate, we have

$$\frac{a_1x_1 + a_2x_2 + \text{etc.}}{A} = \underline{x} \text{ - - - - - - - - - - - where}$$

$\underline{x}$ is the distance to the center of gravity of the section, a_1 and a_2 being the gross areas of the cover plates and angles and x_1 and x_2 being the distance to their centers of gravity. A is the gross area of the entire section. This gives 1.71 in. as the distance of the center of gravity from the top of the cover plates. The

effective depth can now be computed , and is

$$59.25 + (2 \times 15/16) - (2 \times 1.71) = 57.71 \text{ in.}$$

With this depth we proceed to compute the flange section as in the first case. The difference is found to be only 0.2%, and the design will be allowed to stand.

The cover plate next to the angles on the upper flange will extend the entire length so as to exclude the moisture, and the length of the top plate of the upper flange and both plates of the lower flange, will be determined from the curve of Fig. 3. Fig.

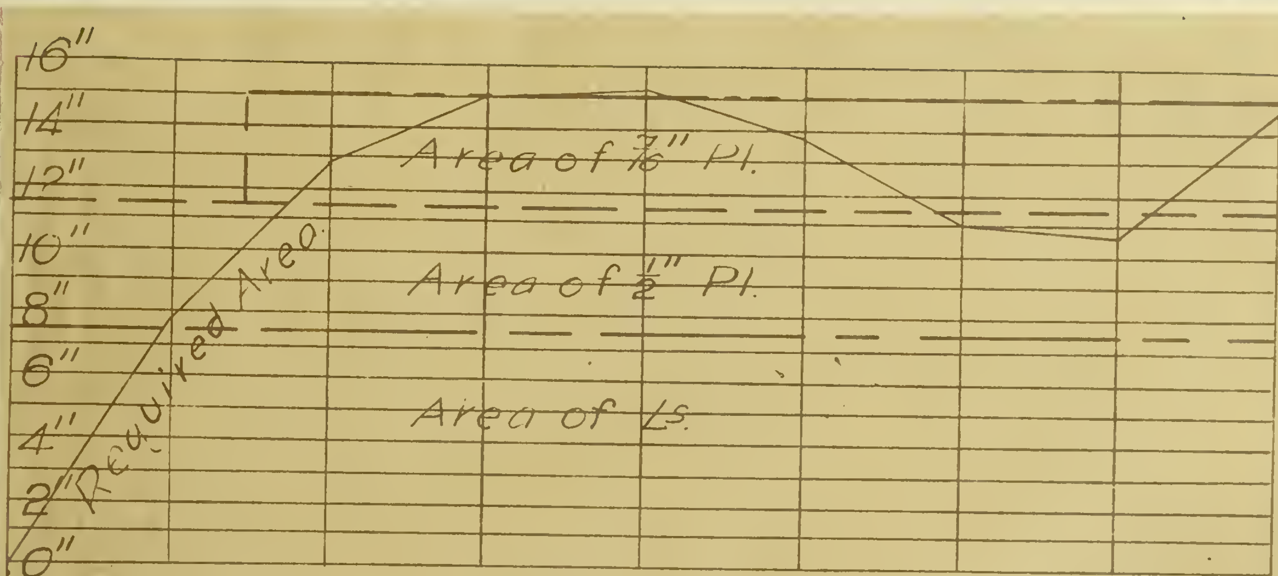


Fig. 3
Curves of Areas.

Fig. 3.

3 is the curve of required flange areas, on which the actual flange areas have been plotted.

Art. 7. The Web.

The stresses in the web of the plate girder are not fully known, but the design is made on the assumption that the web takes all of the shear.

From Table II, the maximum end shear is found to be 85 380 pounds. The allowable shear is 9 000 pounds per sq. in. Dividing this into the total shear, we get 9.49 sq. in. as the required area of the web section. The depth at the end being 48 in., gives a required thickness of 3/16 in., but the specifications limit this thickness to 3/8 in. and so this will be used. Similarly, at the center the required thickness is found to be only 3/16 in. Therefore, a 3/8-inch web will be used throughout.

Owing to the limitations of manufacture, the sizes to which plates can be rolled are well defined. The standard length of a plate 72" x 3/8" is 35 feet. This makes it necessary to splice the web at least twice. On account of the irregular outline of the web, material will be saved by making three splices, one at the center and one six feet from each end of the span. Since the web carries the shear the splice must be sufficient for this purpose.

The maximum shear six feet from the ends is 76 900 pounds. This divided by 9 000 pounds, gives 8.55 sq. in. as the requires net section of the splice plate. The length of the splice plate is 36 in., and the thickness will, therefore be 8.55 ÷ 36 = 0.24 in. The minimum allowable thickness is, however, 3/8 in., and one plate of this thickness will be put on each side of the web. The number of rivets is determined by dividing the shear by 4 920, the allowable bearing of a 7/8-inch rivet in a 3/8-inch plate. This gives 76 900 ÷ 4 920 = 16 as the required number of rivets. It has been determined above that the required thickness at the center is 3/16 in. Therefore, two 3/8-inch splice plates will be used. The number of rivets, determined in a similar manner to that above,

is found to be 24, the bearing in a 3/8-in. plate governing as before.

Art. 8. Rivet Spacing.

The shearing or bearing stress on a rivet in the upper flange of a plate girder is the resultant of two forces, one a vertical force due to the weight of the track and the live load, and the other a horizontal stress due to the difference of flange stresses. Then to find the spacing at any point, it is only required to divide the the allowable shear or bearing value of a rivet by the square root of the sum of the squares of the two forces involved. From the consideration of thesr facts the following formula is derived:-

$$P = \frac{v}{\sqrt{(\frac{V}{h})^2 + w^2}}$$ --------------where

P = the spacing,

v = the allowable value of one rivet,

V = the maximum shear at that point,

h = the effective depth of the girder at that point, and

w = the locallized loading due to the weight of the track and the heaviest wheel load.

As there is no vertical force effecting the rivets in the lower flange, the formula reduces to

$$P = \frac{s\ h}{V}$$

The allowable spacing is, therefore, smmewhat greater than in the upper flange, but the shop cost will be reduced by making it the same. The following curve, Fig. 4, represents the spacing in

as computed for the upper flange. The greatest allowable spacing

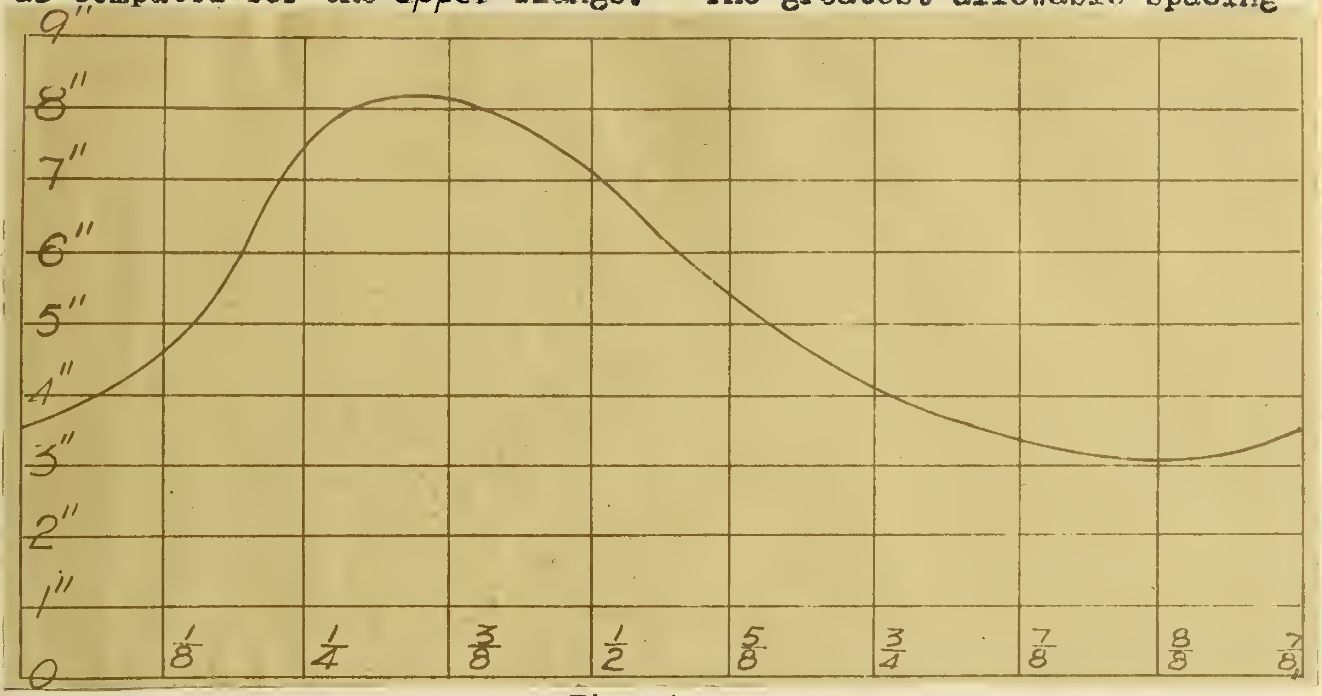


Fig. 4.
Rivet Spacing in Upper Flange.

is, however, six inches.

The number of rivets in the cover plates is determined by dividing the stress which is transferred to the plate by the allowable shearing stress of a rivet. The resulting spacing will be greater than that in the flanges; but it is customary to neglect the computed spacing and to stagger the rivets with those in the flanges.

Art. 9. Stiffeners.

The specifications require that the stiffeners shall be able to withstand the shear without exceeding the unit stress given by the formula $P = 10\,000 - 45\,\frac{l}{r}$. Tho maximum shear at the end is, from Table II, 85 380 pounds. From Cambria we select a 3-1/2" x 3" x 7/16" angle. The radius of gyration of two of these angles placed back to back and separated by the web, is 1.37 inches. The unit stress is, therefore, 8 460 pounds, and the required area

is 10.64 sq. in. This is as near to the required area as it is possible to obtain, and these angles will be used.

Similarly, at the center, designing with 3-1/2" x 3-1/2" x 3/8" angles, the required area is 14.20 sq. in., and the given area 14.94 sq. in. These angles will therefore be used.

At present there is no rational method for the design of the intermediate stiffeners since their stresses and their effect on the web is unknown. They should be able to carry to the web the greatest concentrated load that can come over them. Two angles 3-1/2" x 3" x 3/8" are more than are required, but are the smallest that can be used, and therefore the intermediate stiffeners will be formed of them.

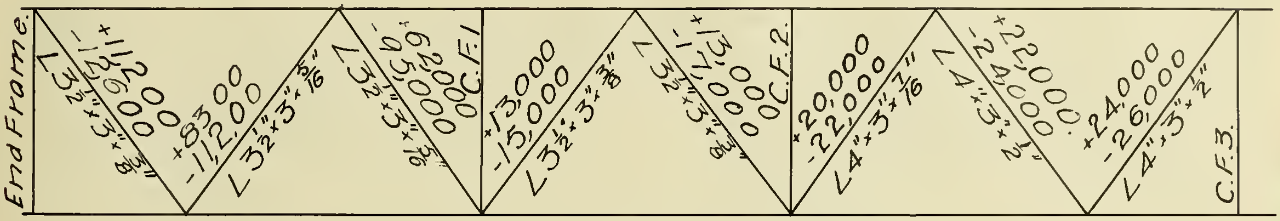
Since the end stiffeners are designed to carry the total end shear, they must have enough rivets to carry the stress to them. Dividing the total shear, 85 300 pounds, by the allowable bearing value of one rivet, 4 920 pounds, gives 17 as the required number of rivets. Similarly at the center support, 24 rivets are required.

Art. 10. Laterals.

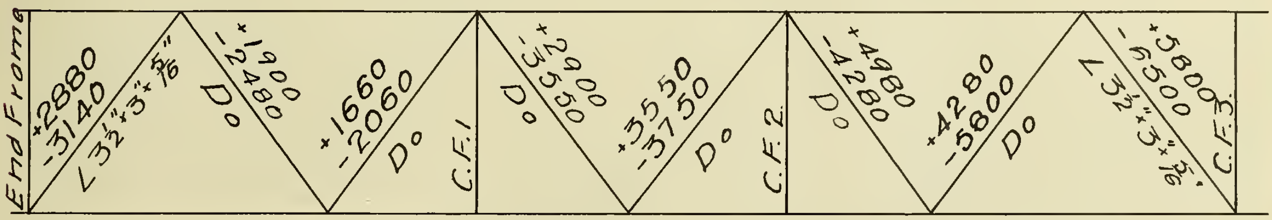
The lower lateral system is to be designed to resist a wind load of 150 pounds per linear foot, and the upper system, in addition to this, is to be designed for a moving load of 450 pounds per linear foot. The loads must be considered as coming from either side in order to obtain the worst conditions. The stresses are computed as for an ordinary Warren truss, and the maxima are shown in Fig. 5 as well as on the strain sheet.

The design, if made for compression, must be also tested for tension. For compression, the unit allowable stress is given

by $P = 13\ 000 - 60\ \frac{l}{r}$, and for tension the unit stress is 18 000 pounds. When an angle in tension is fastened by only one leg, that leg alone is considered as effective.



Upper Lateral System.



Lower Lateral System.

Fig. 5.

For a 3-1/2" x 3" x 5/16" angle, the allowable compressive stress is 7 500 pounds per sq. in., and the maximum wind compressive stress in the lower system is 5 500 puonds. The required area is then 0.72 sq. in., and the actual area is 1.94 sq. in. This angle gives a much larger area than is required, but being the smallest allowable by specifications, it will be used. The required area for tension is 0.32 sq. in., and the net area of a 3-1/2-in. leg is 0.78 sq. in. This sized angle being safe in both tension and compression, will be used throughout the lower lateral system. In a similar manner the angles for the upper system were determined, and are shown on Fig. 5.

Art. 11. Cross-Frames.

The same methods were used for the cross-frames as for the

laterals. The end and center cross-frames will be designed each for half the load of one span, while the intermediate cross-frames will each carry their proportional part. Fig. 6 shows diagrams on which are placed the stresses and the sections used.

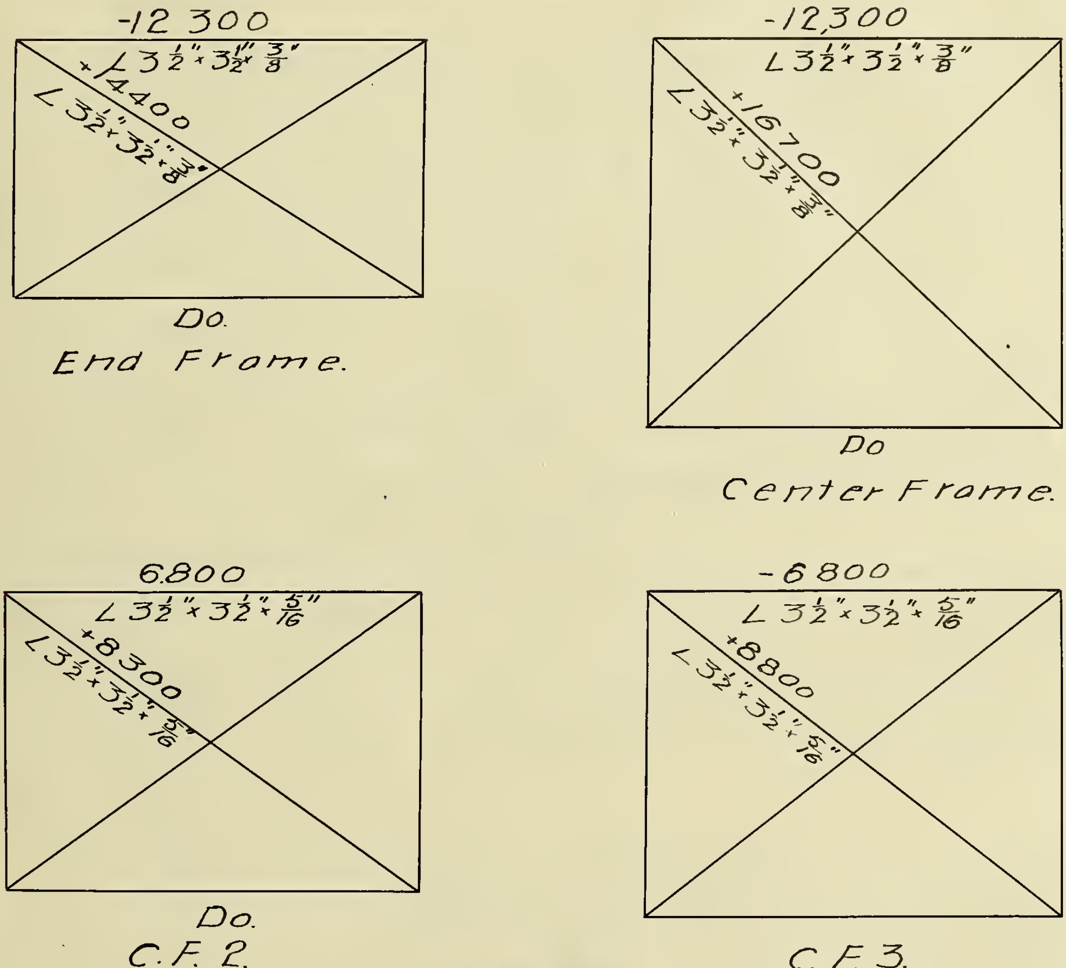


Fig. 6.

Art. 12. Cross-Girders.

When closed, the bridge is supported at the center on wedges, but when swinging, the load is carried by two cross-girders resting on the central post. By making these girders shallow, the point of support will be high and the draw will swing more easily. Their design is similar to that of the longitudinal girders.

The flange stress is 20 250 x 39 ÷ 14 = 56 411 pounds. The allowable unit stress is 3/4 x 20 000 = 15 000 pounds. The

required net flange area is then 56 411 ÷ 15 000 = 3.76 sq. in. The gross area of a 3-1/2" x 3-1/2" x 3/8" angle is 2.94 sq. in. Deducting 0.38 sq. in, for the rivet hole, leaves 2.11 sq. in. as the net area. These angles give a total net area of 4.22 sq. in., and will be used.

The required web area is 20 250 ÷ 9 000 = 2.25 sq. in. To obtain this requires an area of 2.25 ÷ 16 = 0.14 in., but the minimum allowed is 3/8", and this will be used.

Art. 13. Side Braces.

Side braces will be attached to the draw to prevent tipping from unbalanced loads, and to make it more rigid under the passage of trains. Wheels running on a circular track 9'- 6" in diameter will transmit the load to the masonry.

The overturning effect due to a force of 450 pounds per linear foot acting at a distance of six feet above the base of rail, and 150 pounds per linear foot acting at the upper chord, will require an upward force of

$$\frac{(450 \times 80 \times 102) + (150 \times 80 \times 20)}{67} = 58\ 400 \text{ lb. which}$$

acts at the wheel. The stress in the angles at the bottom of the brace will be

$$\frac{58\ 400 \times 18}{60} = 17\ 520 \text{ lb.}$$

The required net area of these angles is then 17 520 ÷ 16 000 = 1.10 sq. in. The upper angles will have a stress of 58 4000 x 1.09 = 63 500 pounds. The allowable stress computed from the compression formula is 11 600 pounds. This is gotten by using a radius of gyration of 1.61, which is that of two 3-1/2" x 3-1/2" x 3/8" angles placed back to back and 3/8 inches apart. The general arrangement

of the brace, as well as its sizes is shown in Fig. 7.

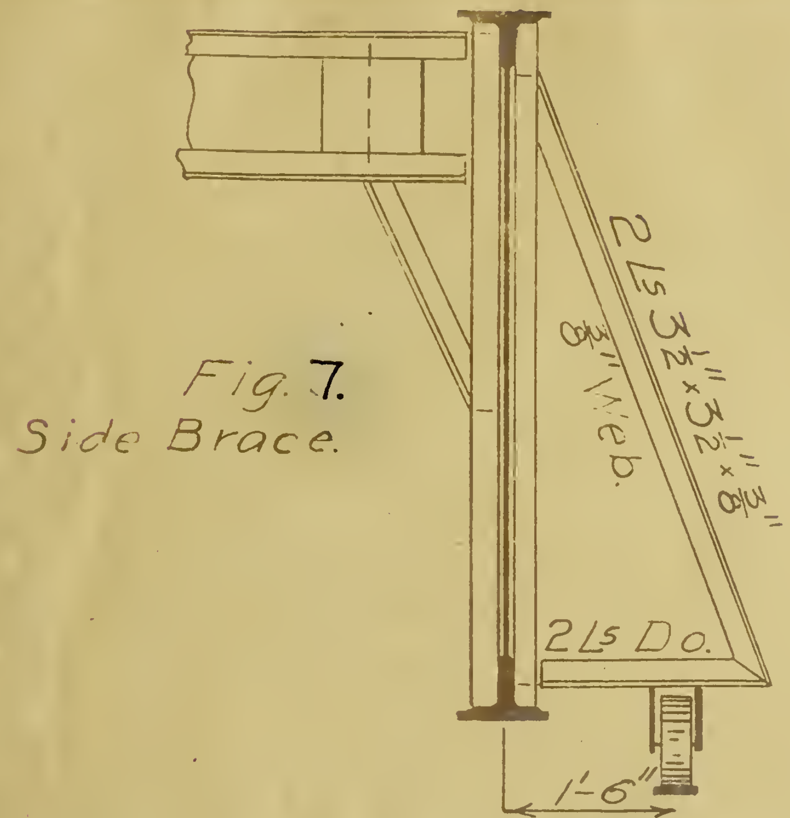


Fig. 7.

Art. 14. Center Bearings.

The center bearing bearing carries the dead load only, as the live load is transferred to the masomry by means of the side braces. To transfer this load to the masonry at 250 pounds per sq. in., would require the base of the post to be only twenty inches in diameter. As this is too small to give the required stability, consideration of this latter factor will govern the design. Fig. 8 gives a general detail of the post. As metals of the same kind do not, as a general rule, work well when in bearing upon one another, the bearing surfaces are made of a spherical surfaced disk, and this bears on a phosphor bronze plate so

shaped as to give a complete bearing. In order to save matal, the

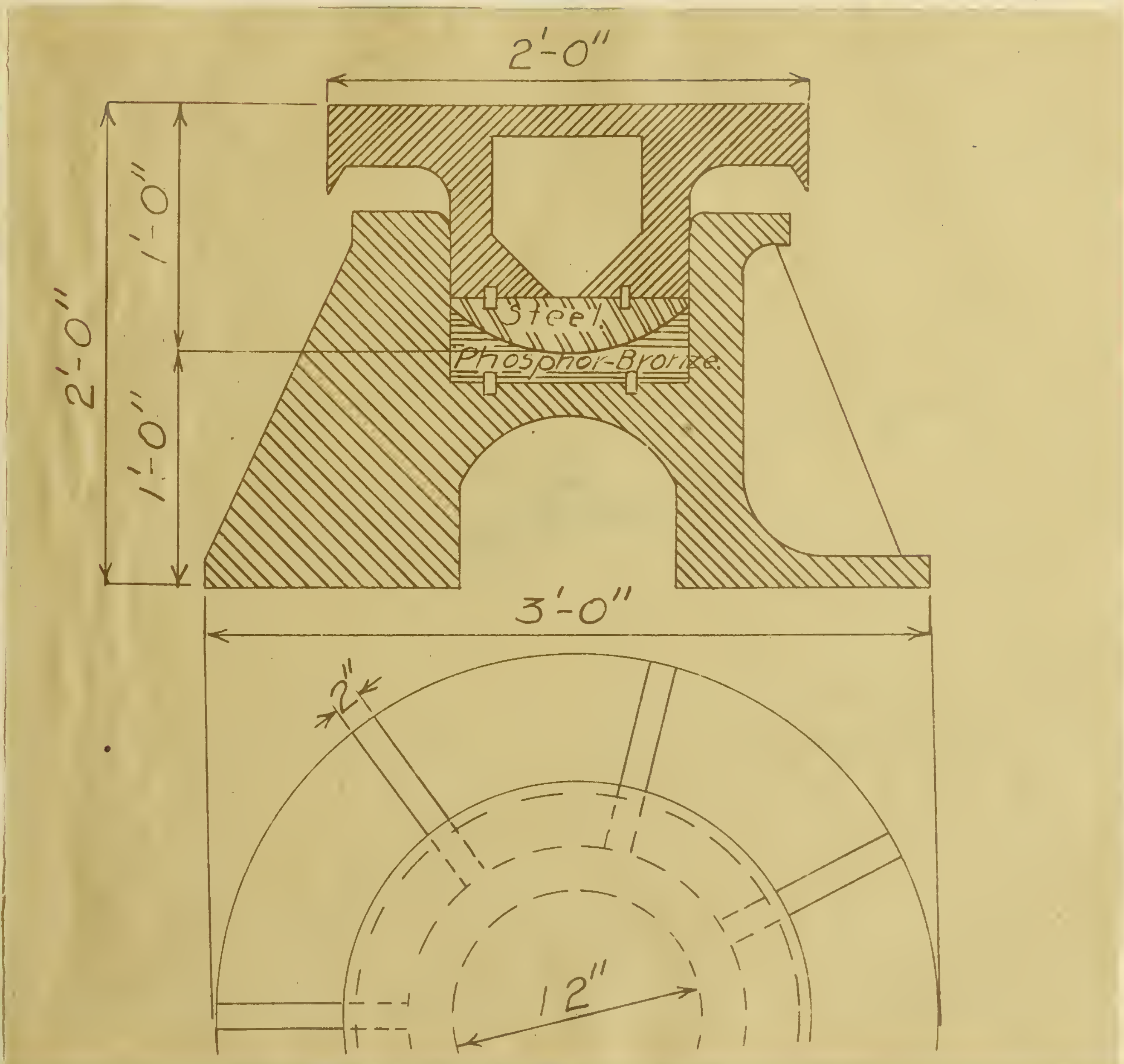


Fig. 8.

center is hollow and the sides are reinforced by braces cast in place.

Art. 15. End Deflections.

The end deflections must be known in order to compute the distance that the wedges have to be driven. For a cantilever having the shape of this draw, the end deflections are given by the

formula

$$D = \frac{1.166\ Wl^3}{Eh^3}$$

$$= \frac{W\ l^3}{161\ 500\ 000\ I} \text{ - - - - - - - - - - where}$$

W = the total uniform load,

l = the length of arm in inches,

E = the modulus of elasticity of steel, and

I = the moment of inertia at the center support.

The moment of inertia is computed as follows:-

Cover plate = $2(9.37 \times 37.02^2)$	= 25 626 in.
Angles = $2(9.50 \times 35.10^2) + 4 \times 19.26$	= 23 506 "
Web = $1/12 \times 3/8 \times 72^2$	= 10 720 "
Total	= 59 952 in.

Substituting the required values in the formula for the deflection, there results

$$D = \frac{20\ 250 \times 480^3}{161\ 500\ 000 \times 59\ 952} = 0.24'' = 1/4''$$

The maximum negative reaction for a static load is R = $- 1/16\ Wl = - 3770 \times 40 \div 16 = - 9\ 430$ pounds. The additional negative reaction due to the impact will be quite large. Since Cooper does not give an impact formula, the one used by the American Bridge Co. will be taken. It is

$$I = \frac{300}{L + 300} \text{ - - - - - - - - - - - - - where}$$

S = the static stress,

L = the loaded length in feet, and

300 = a constant, the same for all conditions.

Substituting in this formula we derive the value of the impact stress which is (9 430 x 300) ÷ (40 + 300) = 8 300 pounds. The sum of these two give 17 730 which is the negative reaction.

The deflection of a girder of uniform cross-section, loaded at the end is

$$D = \frac{P\ l^3}{3\ E\ I}$$

This cannot be directly applied to the case at hand as the decrease in depth at the ends increase the deflection. Wright, in his text on draw spans, modifies the formula to

$$D = \frac{P\ l^3}{1.9\ E\ I}$$

P is the concentrated load, and the other factors are the same as for uniform load. Using this formula and substituting our values in it we get

$$D = \frac{17\ 730 \times 480^3}{1.9 \times 29\ 000\ 000 \times 59\ 952} = 0.59''$$ as the deflection at the end. Both ends will be raised 0.59 ÷ 2 = 0.295" which will be sufficient to keep the span from hammerimg.

Art. 16. Wedges.

The maximum reactions require that the end masonry plates have areas of 324, and those at the center 754 sq. in. The former will be 16" x 24" and the latter will be 24" x 31-1/2". The reactions are transferred to these plates by the wedges. Those at the center are driven only to a firm bearing, while those at the ends must raise the ends sufficiently to take out the deflection and create an upward pressure as great as the negative reaction due to the live load, in order to prevent "hammering". Fig. 9 and 10

show the design of the wedges. To determine the shape of the sur-

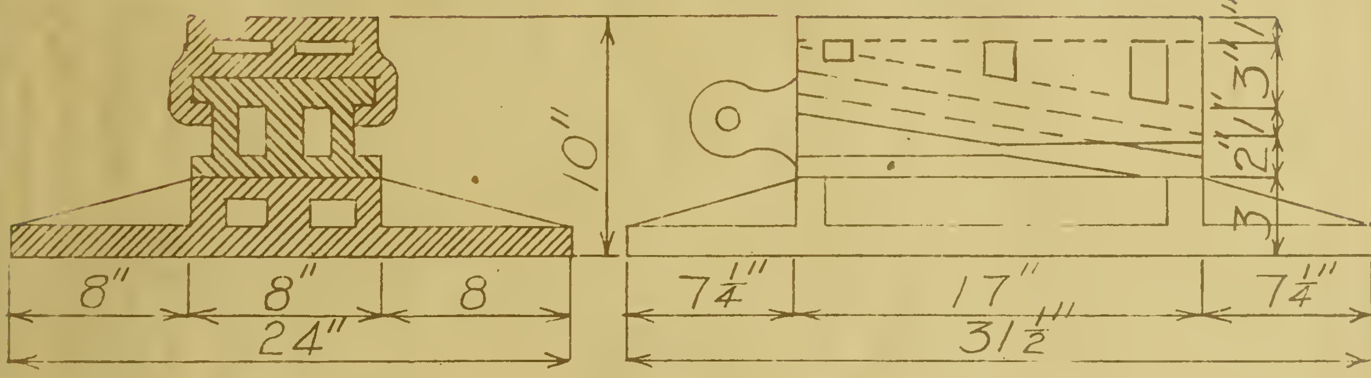
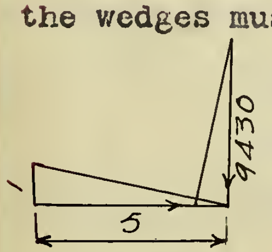


Fig. 9.
Center Wedges.

Fig. 9. Center Wedges.

face, considering a throw of 10 inches, a clearance of 1-1/2 inches plus the deflection of 0.59 inches requires about 2 inches. The slope will then be 2/10 = 1/5.

The total maximum negative reaction is 17 725 pounds, and the wedges must exert an upward force equal to this amount. From



the similar triangles, it is seen that the horizontal force must be one-fifth of 17 725 pounds plus the force due to friction. The coefficient of friction of metal on metal varies from 0.05 to 0.08 for well lubricated surfaces, to 0.15 to 0.24 for dry surfaces. If we consider the coefficient of friction to be 0.10, the total force required to operate the wedges will be 3 540 pounds. This force will not be required

throughout the entire motion of the wedge, but will be zero until

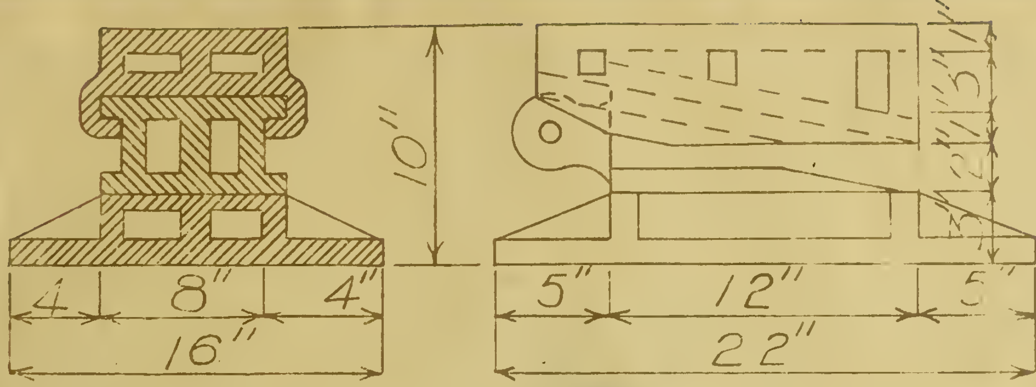


Fig. 10. End Wedges.

the beginning of the last inch of the thrust, and will then increase to the amount which will be the maximum required.

PART III. THE MACHINERY.

Art. 17. The Inertia of the Span.

The machinery for turning the draw must be designed to overcome the following forces:-

a. The inertia of the span,

b. The friction of the center bearing,

c. The friction of the balance wheels, and

d. The friction of the gearing and shaft bearings.

These forces will now be considered in detail.

The inertia of the span is greater in amount than any of the others, and will be considered first. Let the time required to opent eh draw be 1-1/2 minutes. In that time, a mass equal to the weight of the bridge must be started from rest and revolved 90 de-

grees. The first half of the motion will be an acceleration and the last half will be a retardation. Throughout the motion, the force required to accelerate or retard is,

$$P = \frac{2\ I\ \theta}{R\ t^2}$$ - - - - - - - - - - - where

P = the required force,

I = the moment of inertia of the span about the center,

θ = the angle expressed in radians = $\frac{\pi}{4}$,

R = the radius of the rack circle = 4 ft., and

t = the time required to swing = 45 seconds.

The moment of inertia is readily computed, and is,

$$I = \frac{W\ (l/2)^2 + (b/2)^2}{g}$$

$$= \frac{81\ 000(20.25^2 + 3.25^2)}{32.2}$$

$$= 1\ 050\ 000$$

Substituting the required values in the first formula above, we have,

$$P = \frac{2 \times 1\ 050\ 000 \times 3.1412}{4 \times 45^2 \times 4}$$

$$= 204 \text{ pounds.}$$

Art. 18. Friction of the Center Bearing.

The force required to overcome this friction is practically

$$F = \frac{0.66\ f\ P\ r}{R}$$ - - - - - - - - - - -where

f = the coefficient of friction = 0.005,

P = the weight of the dead load,

r = the radius of the shaft, in inches, and

R = the radius of the rack circle, in feet.

Substituting the required values in the formula, we have

$$F = \frac{0.66 \times 0.005 \times 81\ 000 \times 0.5}{4}$$

= 34 pounds as the force necessary to overcome the friction caused by the dead load on the center bearing.

In addition to the above, there is another cause of friction at the center bearing. This is caused by the horizontal thrust of the wind. A wind force of 30 pounds per sq. ft. of girder surface will cause a horizontal thrust of 12 100 pounds. To overcome this, we find, by substituting in the above formula, that a force of 151 pounds is necessary. In this case we use a coefficient of 0.10 and a constant of unity instead of 0.66.

Art. 19. Friction of the Balance Wheels.

The weight on the balance wheels will be due to an unbalanced load, and the friction will be rolling friction. If we assume a load of 1 000 pounds at 20 feet from the center, the weight on the balance wheels will be 1 000 x 20 ÷ 4.75 = 4 200 pounds. The coefficient of rolling friction for steel on steel varies from 0.003 to 0.020. It will be assumed as 0.010. Then the force applied at the rack circle to overcome this resistance is,

$$P = \frac{f\ W\ R_1}{r\ R_2} \text{ - - - - - - - - - - - - - where}$$

f = the coefficient of friction = 0.010,

W = pressure on wheel, due to unbalanced load = 4 200 lb.,

R_1 = radius of wheel circle = 4.75 ft.,

R_2 = radius of rack circle = 4.00 ft., and

r = radius of balance wheel = 0.50 ft.

Substituting the values as indicated, in the formula gives us 100 pounds as the force required to overcome this resistance.

The friction of the bearings of these wheels must now be considered. The coefficient of friction will be assumed as 0.10, the force required to overcome this resistance being given by the formula

$$P = \frac{f\ W\ r}{R} \text{ - - - - - - - - - - - - where}$$

f = the coefficient of friction = 0.10

W = pressure on the wheel = 4 200 lb.,

r = radius of the wheel axle = 0.75 in., and

R = the radius of the wheel = 6 in.

Substituting, we have,

$$P = \frac{0.10 \times 4\ 200 \times 0.75}{6}$$

= 52 pounds as the required force applied at the rack circle to avercome the friction of the balance wheels in their axle boxes.

If the unbalanced wind force of 1 500 pounds acts at the same time, the force required to overcome the resistance due to it will be directly proportional to the forces in the two previous cases, and will be(1 500 x 100) ÷ 4 200 = 35 pounds and (1 500 x 52) ÷ 4 200 = 18 pounds respectively.

Art. 20. Friction of the Gearing.

The friction of the gearing cannot be computed before the design is made. However, it will be only a small per cent of the other forces and might be neglected without causing any material difference in the design. It will not be neglected, but will be as-

sumed as 25 pounds, and should this prove to be greatly in error, the design will changed so as to correspond to the proper amount.

Art. 21. Power Required for Operating.

The addition of these factors gives 619 pounds as the force applied at the rack circle, which is required to operate the draw in the given time. The maximum velocity at which this force is applied is $(2 \times 8 \times \pi) \div 8 \times 45 = 0.14$ feet per second. Hence $619 \times 0.14 \div 550 = 0.15$ horse power is required.

These computations might have been shortened by the application of one of the empirical formulas which have been derived from experiments. As an example, we will apply the formula

$$H.\ P. = \frac{0.0125\ W\ \pi\ D}{550 \times 4\ t} \text{ - - - - - - - - - - - where}$$

D = the diameter of the rack circle in feet, and

t = the assumed time of turning 90 degrees.

Substituting,we get

$$H.\ P. = \frac{0.0125 \times 81\ 000 \times 3.1416 \times 8}{550 \times 4 \times 90} = 0.133$$

On account of the assumptions which it was necessary to make, this is undoubtedly as reliable as the more laborious process which was used.

A man can easily push with a force of 75 pounds. Then to create a force of 619 pounds will require a multiple of $619 \div 75 = 8.3$ times. If the power is applied at the end of a 3.5-foot lever, a wheel of $(3.5 \times 12) \div 8.3 = 5.0$ inches radius will be required. The general arrangement of the shaft which is to be operated by manual labor, as well as the dimensions which are determined latter,

are shown in Fig. 11.

Art. 22. Design of Machinery.

The shafting and gearing will necessarily be designed for the greatest stress which will probably come upon them. Experiment has taught that men in charge of draw bridges are careless, and exert their utmost strenght before investigating to see if there is anything to prevent the draw from turning. If 200 pounds is applied at the end of the 3.5-foot lever, the shaft which transmits the power to the gearing should have a diameter of

$$d = \sqrt[3]{\frac{16\ P\ r}{\pi\ s}} \text{ - - - - - - - - - - - - - - -where}$$

P = the applied force in pounds,

r = the distance at which it is applied in inches, and

s = the allowable stress per sq. in. = 15 000lb.

Substituting, we get

$$d = \sqrt[3]{\frac{16 \times 200 \times 42}{3.1416 \times 15\ 000}} = 1\text{-}7/16 \text{ inches.}$$

Since it is necessary to connect the shaft to the gearing by a key, the diameter will be made 1-5/8 inches.

The pitch of the gear teeth is computed from the formula $k = 0.025(0.67\ P)^{\frac{1}{2}}$, where P is the total tooth pressure = 200 x 8.3 = 1 660 pounds. The pitch is therefore, $0.025(0.67 \times 1\ 660)^{\frac{1}{2}}$ = 0.83 inches. A pitch of 7/8 inches will be used. The length of a tooth is 2.5 x 0.875 = 2.00 inches, and the number of teeth in the wheel base will be (10 x 3.1416) ÷ 7/8 = 36.

The rack will necessarily be cast in sections, and each section will have the same number of teeth. Four sections will be used, and hence the length of the pitch circle must be divisable by

4 x 7/8 = 3.5 inches. The radius is 8 feet, and the circumference is, therefore, 311.5 inches. This gives 89 teeth in each section. Fig. 11 is a detail of the operating mechanism.

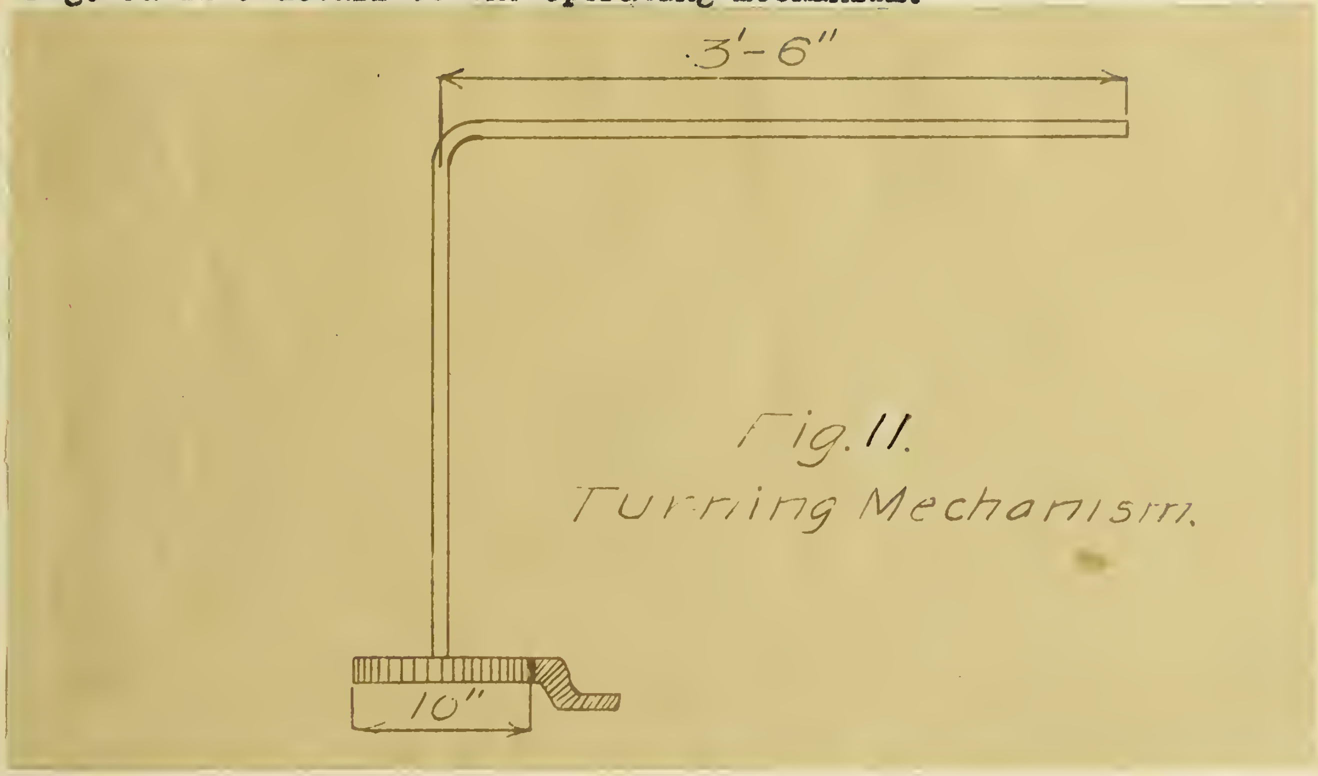


Fig. 11.

We may now compute the power necessary to overcome the friction of the shaft and gearing. A coefficient of 0.07 gives 2(619 x 0.07) = 86 pounds. Our assumption is thus seen to be too small. The force at the end of the lever, which is required to overcome this is (43 x 13/16) ÷ 42 = 0.8 pounds for the shaft and (43 x 2.5) ÷ 42 = 2.5 pounds for the gearing. This is two pounds more than the assumed force, and gives a total of 77 pounds at the end of the lever. As this is well within the limit of a man's power, no change will be made in the computations and the design.

The design of the wedges will now be undertaken. It was determined that the force necessary to drive one wedge was 3 540 pounds, and that this is required just at the end of the thrust.

If we consider that this force is the force required when the wedge still has 1/4 inch to move, we will be providing for a possible emergency. The resulting power required may now be computed algebraically, or determined graphically as in Fig. 12. AC = 3 540 is

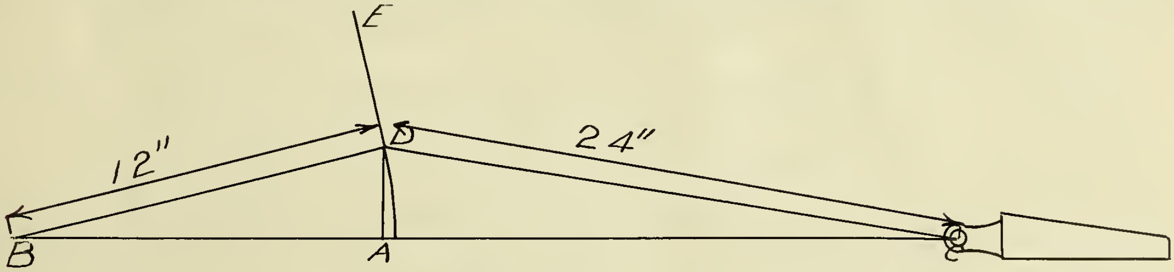


Fig. 12.

replaced by DC = 24/23.75 x 3 540 = 3 600 pounds. We now have the forces DB. DE. and DC in equilibrium. Then DE ÷ sin.BDC = DC ÷ sin.BDF or DE ÷ 0.246 = 3 600 ÷ 1, whence DE = 885 pounds as the force at D perpendicular to DB, which is necessary to give a horizontal force of 3 540 pounds. A coefficient of friction of 0.07 requires a force of 0.07 x 3 600 = 250 pounds to be added to the above, making a total of 1 135 pounds. To drive all four wedges will require 4 540 pounds. The center wedges are merely to be driven to a firm bearing, and 1 000 pounds should be sufficient for this purpose.

Since the exertion is to be of short duration, we may assume the operator to push with a force of 120 pounds. It will then be necessary to multiply this 5 540 ÷ 120 = 46.2 times. The mechanism shown on the strain sheet produces 56 multiplications, and requires 5 540 ÷ 56 = 99.0 pounds at the end of the lever arm.

The friction of the driving shaft is 99.0 x 42/3 x 0.07 = 97 pounds, and the force necessary to overcome this is 97.0 x 13/16 ÷ 42 = 2 pounds. The shaft will be made 1-5/8 inches in diameter,

since we have the same conditions as in the turning machinery. Assuming the same sized shaft for the other bearings, the force required to overcome their friction will be the same as that for the first case, since the increase in pressure is directly proportionally to the multiplying power of the gearing. The total resistance from the friction of the shafting will then be 3 x 2 = 6 pounds.

Assuming the same coefficient of friction for the gearing, the force necessary to overcome the friction of the first gearing is 3 x 16/13 x 2 = 7 pounds, which may also be assumed as that required for the bevel gears. The total force now required at the end of the lever is 99.0 + 6 + 14 = 119 pounds which is within the limit allowed.

The shaft which drives the wedges has two stresses acting on it, one due to bending, and one due to torsion. Consider a shaft 3 inches in diameter. The stress due to bending is

$$S = \frac{3\ 700 \times 5/6 \times 12 \times 1.5 \times 64}{81 \times 3.1416} = 13\ 500 \text{ lb. per sq. in.}$$

and that due to torsion is

$$S = \frac{16 \times 1\ 850 \times 12}{27} = 13\ 000 \text{ lb. per sq. in., making a}$$

total of 26 500 pounds per sq. in. As this is not excessive in shafting operated by hand, this diameter, 3 inches, will be used.

The deflection of the end of the shaft farthest from the turning point is given by the expression

$$\theta = \frac{57.3\ P\ r\ l}{F\ J} \text{ - - - - - - - - - - -where}$$

P = the twisting force in pounds,

l = the distance from the center of its point of application.

F = the shearing modulus of elasticity, and

J = the polar moment of inertia.

Substituting, we have

$$\theta = \frac{57.3 \times 1\ 850 \times 12 \times 62 \times 32}{12\ 000\ 000 \times 81} = 0.8 \text{ degrees.}$$

The deflection is directly proportional to the length, and therefore the opposite end has a deflection of $16/62 \times 0.8 = 0.2$ deg. = 36 min. This is not enough to prevent simultaneous action of the wedges, and the shaft is satisfactory in this respect.

The latches are required to stop the draw in the proper position to allow the driving of the wedges, and to hold it against the wind. The device shown on the strain sheet is intended to latch automatically. The rod carrying the wheel at its end is raised and turned 90 degrees. It is then held up by the projection (shown dotted on the strain sheet) which rests on the top of the angle. As the draw swings, the rod which projects at right angles to the rod strikes a projection on the abutment and turns back, allowing the rod to fall into its former position.

Art. 23. Character of Operating Power.

There are many sources of power in use for the operating of draw spans, but for small spans, there are in common use three kinds, namely, electric motors, gasoline engines, and manual labor.

Electricity is the best form of power. It is safe, reliable, and easily applied. The gasoline engine is comparatively cheap and easily operated, but, owing to its liability to get out of order, it requires an operator with some mechanical skill. A draw the size of the one in question can easily be operated by one

man, and since an operator is required in any case, it will be economical to use manual labor since it will effect a large saving in the cost of machinery which would be in use only a part of the time.

PART IV. COST.

Art. 24. Estimate of Weight.

A detailed estimate of the weight will now be made in order to verify the original estimate, and to give a basis for the estimate of cost. The section and length of each piece of steel is taken from the strain sheet and the weight per linear foot from the Cambria handbook. Table VI is self-explanitory. The preliminary estimate gave 48 600 pounds as the total weight. The computation gives 45 848 pounds for the weight of steel in the bridge proper, and 1 929 pounds for the weight of the machinery, making a total of 47 770 pounds or 98 per cent of the estimate.

Table VI.

Computation of Weights.

Name.	No.	Section.	Length.	Wt. per Ft.	Total Wt.
1	2	3	4	5	6
		Top Chord.			
L^s	4	$6'' \times 4'' \times \frac{1}{2}''$	81.0	16.20	5 249
Pls.	2	$10'' \times \frac{3}{8}''$	81.0	12.75	2 060
Pls.	2	"	64.0	12.75	1 635
					8 944
		Bottom Chords.			
L^s	4	$6'' \times 4'' \times \frac{1}{2}''$	81.2	16.20	5 262

Table VI. (cont'd.)

1	2	3	4	5	6
Pls.	2	$10'' \times \frac{3''}{8}$	79.0	12.75	2 010
"	2	"	64.0	12.75	1 635
					8 907
			Webs.		
Pls.	2	$\frac{3''}{8}$	Sketch.		12 332
L^s	16	$3\frac{1''}{2} \times 3'' \times \frac{7''}{16}$	4.0	9.10	580
L^s	8	$3\frac{1''}{2} \times 3'' \times \frac{3''}{8}$	4.0	7.90	254
"	8	"	4.4	"	279
"	8	"	4.7	"	297
"	8	"	5.0	"	317
"	8	"	5.3	"	336
"	8	"	5.6	"	355
"	8	"	5.9	"	374
"	16	$3\frac{1''}{2} \times 3\frac{1''}{2} \times \frac{3''}{8}$	6.0	8.50	816
Pls.	16	$4'' \times \frac{1''}{2}$	3.0	6.80	326
"	4	"	3.7	"	101
"	4	"	4.3	"	118
"	4	"	5.0	"	136
"	4	$24'' \times \frac{1''}{2}$	5.0	40.80	815
"	8	$11'' \times \frac{3''}{8}$	3.0	14.00	336
"	8	$4'' \times \frac{1''}{8}$	3.0	1.70	41
					17 836
		Upper Lateral System.			
L^s	6	$3\frac{1''}{2} \times 3'' \times \frac{5''}{16}$	7.0	6.60	277
"	4	$3\frac{1''}{2} \times 3'' \times \frac{3''}{8}$	7.0	7.90	222
"	2	$4'' \times 3'' \times \frac{7''}{16}$	7.0	9.80	138
"	4	$4'' \times 3'' \times \frac{1''}{2}$	7.0	11.10	155

Table VI. (cont'd)

1	2	3	4	5	6
Pls.	18	$17'' \times \frac{3}{8}''$	2.5	21.7	980
"	8	$12'' \times \frac{3}{8}''$	0.9	15.3	110
					1 882
		Lower Lateral System.			
L^s	16	$3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$	7.0	6.6	730
Pls.	18	$12'' \times \frac{3}{8}''$	2.0	15.3	550
"	8	$9'' \times \frac{3}{8}''$	1.0	"	110
					1 490
		End Frames.			
L^s	4	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	7.0	8.5	223
"	4	"	5.8	8.5	199
Pls.	8	$18'' \times \frac{3}{8}''$	1.5	23.0	275
"	2	$8'' \times \frac{3}{8}''$	0.7	10.2	14
					726
		Intermediate Cross -Frames.			
L^s	4	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{16}''$	7.2	7.2	210
"	4	"	7.6	"	230
"	8	"	5.8	"	334
"	4	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$	8.0	8.5	262
"	4	"	5.8	"	198
Pls.	24	$18'' \times \frac{3}{8}''$	1.5	23.0	825
"	10	$8'' \times \frac{3}{8}''$	0.7	10.2	70
					2 129
		Cross Girders.			
L^s	8	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$	6.0	11.1	534
"	8	$3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{16}''$	2.0	6.6	106
Pls.	4	$8'' \times \frac{3}{8}''$	6.0	10.2	245

Table VI. (cont'd.)

1	2	3	4	5	6
Pls.	2	16" x $\frac{3}{8}$"	3.5	20.4	143
"	4	18" x $\frac{3}{8}$"	"	23.0	323
"	8	10" x $\frac{3}{8}$"	1.0	12.8	102
					1 453
		Side Braces.			
L^s	8	$3\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{3}{8}$"	4.2	8.5	285
"	8	"	1.8	"	123
Pls.	4	12" x $\frac{3}{8}$"	4.0	15.3	245
"	8	6" x $\frac{3}{8}$"	1.5	7.7	92
					745
Adding 4 per cent of the above weights for rivet heads=					1 763
				Grand Total =	45 848 lb.

Machinery.

Shafting - - - - - - - - - - 700 lb.

Castings - - - - - - - - - -1 229 "

Center Post etc. - - - - 3 822 "

Total = 6 751 lb.

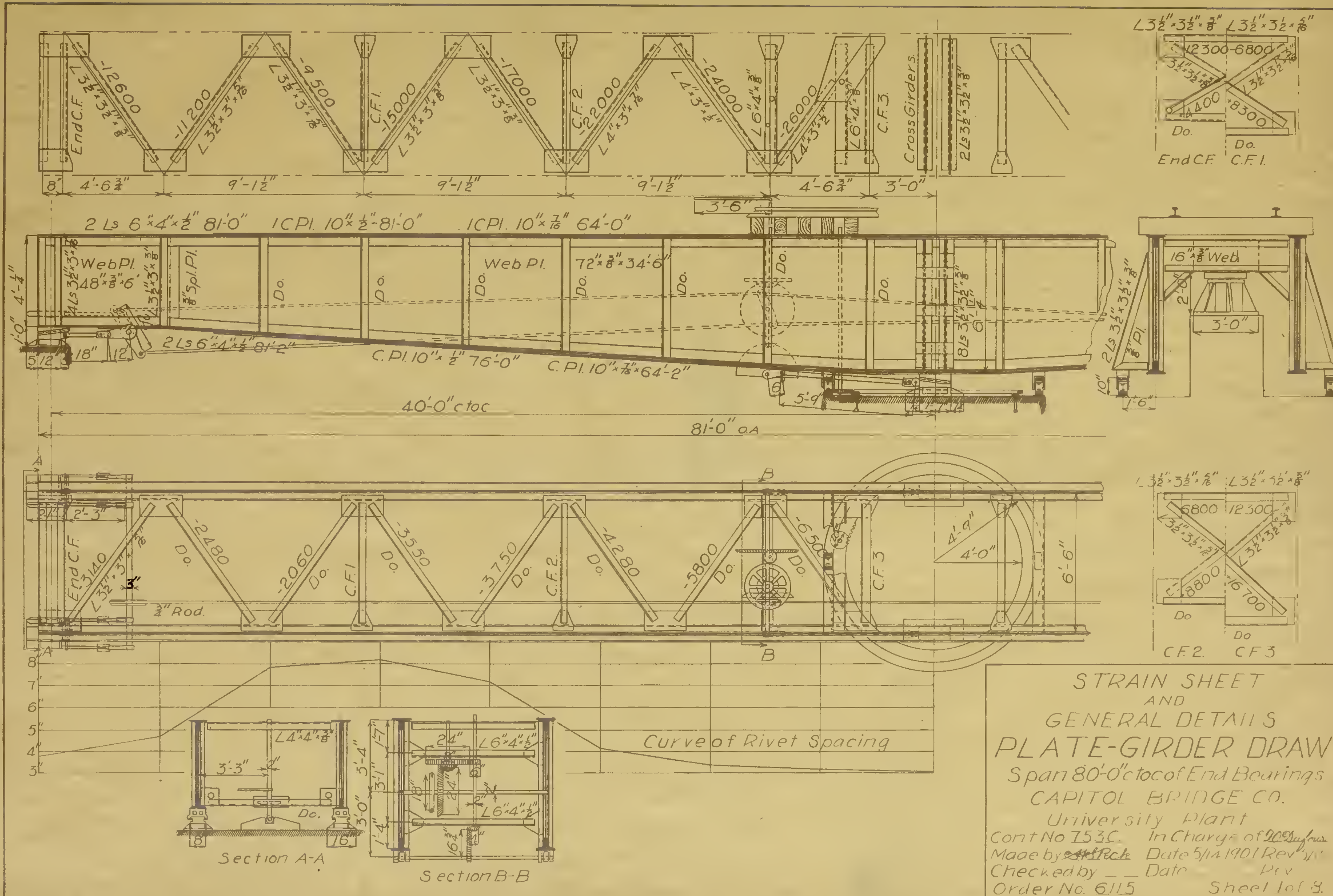
Art. 25. Estimate of Cost.

The cost of steel bridges is estimated by the weight of the material used, and therefore, for structures closely similar, the cost varies almost directly with the weight. In Table VII, the weights are taken fron Table VI. The prices of steel are from the "Iron Age". They are the Pittsburg quotations for April 11, 1907. The price of gearing was furnished by Foot Bros., 24-30 S. Clinton St., Chicago, Illinois.

Table VII.

Estimate of Cost.

Name.	Amount.	Price per Unit.	Cost.
Angles.	18 360 lb.	1.70 ¢	$ 312.12
Plates.	25 625 "	1.75 "	448.43
Rods.	90 "	1.84 "	1.46
Shop Cost.	45 848 "	0.75 "	343.86
Paint.	23 gals.	$1.00	23.00
Gearing, Cast arms, & Rack.			97.00
Shafting.			28.20
Wedges.	1 752 lb.	5.00 ¢	77.60
Center Post.			194.50
Balance Wheels.			8.00
Track.			12.00
Bolts.	180 lb.	2.27 ¢	4.09
Drafting.	25.3 tons.	$4.00	101.20
Freight, Pittsburg to Urbana.	25.3 tons.	23.00 ¢	59.19
Erection.	25.3 tons.	$15.00	379.50
			$2 088.55
Engineering.	5 per cent of the cost - - - - -		104.43
		Total =	$2 192.98


STRAIN SHEET
AND
GENERAL DETAILS
PLATE-GIRDER DRAW
Span 80'-0" c to c of End Bearings
CAPITOL BRIDGE CO.
University Plant
Cont No 753C. In Charge of
Made by Date 5/14 1901 Rev
Checked by Date Rev
Order No. 6115 Sheet 1 of 8.
Cross Girders.
End C.F.
C.F. 1.
C.F. 2.
C.F. 3.
Web Pl.
Curve of Rivet Spacing
Section A-A
Section B-B
40'-0" c to c
81'-0" o.a.